DESIGN OF REINFORCED CONCRETE ARCH HIGHWAY BRIDGE

BY

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ARMOUR INSTITUTE OF TECHNOLOGY

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DESIGN OF REINFORCED CONCRETE ARCH HIGHWAY BRIDGE

A THESIS

PRESENTED BY

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- Preface -

In compiling this thesis, it has been the aim of the author to present and solve, in a systematic manner, a problem that might confront a young civil engineer.

The problem is the design of a reinforced concrete arch highway bridge over the Fox River at Montgomery, Kane County, Illinois.

The type of arch that was decided upon was one with fixed ends and arch ribs. The hingeless arch was selected because of its rigidity and also on account of rock being found near the surface.

Ribbed construction was adopted because it is light and also requires no waterproofing.

The author is under obligation to Mr. R. W. Kinnally, Professor J. C. Penn and to Professor Stanley Dean for help and for many valuable suggestions.

Jos. L. Duffy

Chicago, Ill. May 1915.

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Introduction

The work is essentially divided into two parts. In Chapter I is given the specifications and the formulas used in the design. In Chapter II is given the complete analysis of the arch, abutment, pier and floor system, which is called the deck. The method of arch analysis presented here is based on the elastic theory as given in the work, "Principles of Reinforced Concrete Construction" by Turneaure and Maurer (Copyright, 1907, 1909 by F. E. Turneaure and E. R. Maurer.)

The author has assumed a knowledge of the formulas used in the design and therefore the derivation of such will not be given.

Jos. L. Duffy

Chicago, Ill. May 1915.

Chapter I

SPECIFICATIONS
for
REINFORCED CONCRETE ARCH
HIGHWAY BRIDGE

- o -

"The most perfect system of rules to insure success must be interpreted upon the broad grounds of professional intelligence and common sense." -Cooper

The Second Report of the Joint Committee on Concrete and Reinforced Concrete presented in the Proceedings of the American Society of Civil Engineers, February 1913 shall be assumed as incorporated in these specifications.

Specifications given here shall govern, when articles relating to similar subjects differ in requirements.

Jos. L. Duffy

LOADS

Dead Load

1 This includes the weight of the structure and fixed loads and forces.

Weight of Materials

2 In computing the dead load, the following unit weights shall be used:

Steel	490	lbs./cu.ft.
Concrete	150	"
Earth fill	110	"
Macadam and gravel rolled	140	"
Yellow pine and Douglas fir	42	"
Oak	54	"

Live Loads

3 This consists of a uniform live load of 125 pounds per square foot of roadway, and 100 pounds per square foot over sidewalks, or the concentrated live load of 20 tons indicated in Figures I (a). In all cases, the load used and its position on the bridge shall be that which will produce the greatest stress in that part of the structure under consideration.

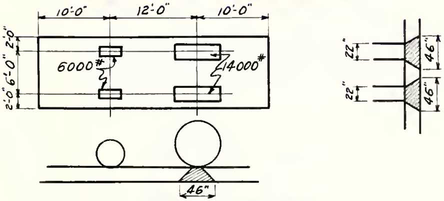
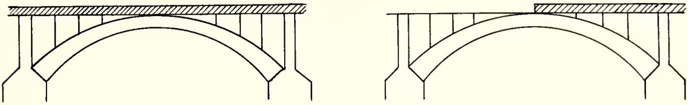
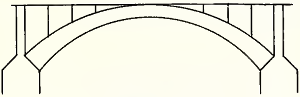


Figure I (a)
Concentrated Loading



FULL L.L. AND D.L.

FULL D.L.
L.L. ON ONE-HALF ONLY



D.L. ONLY

Figure I (b)
Combinations of Loadings for Arch Ribs

Note: In all cases where live load is called for, uniform load shall be used and allowance included for impact.

Impact

4 The stresses due to the dynamic effect or impact of live load shall be added to the maximum computed live load stresses and shall be determined by taking 40% of the maximum live load stresses. Impact shall not be added to strains produced by lateral or wind forces.

Wind Load

5 The wind load shall consist of a moving load of 300 pounds per lineal foot on loaded chord, plus 30 pounds per square foot on the loaded structure.

ALLOWABLE STRESSES

6 The modulus of elasticity of steel shall be taken as 30,000,000 pounds per square inch.

7 The modulus of elesticity of concrete shall be taken as 2,000,000 pounds per square inch.

8 The coefficient of expansion of concrete, plain or reinforced, shall be taken as 0.000006

CONCRETE ABUTMENTS AND PIERS

9 Tension in concrete	00	lbs.
Compression, class B concrete	450	lbs.
Tension, reinforcing steel	16000	lbs.
Bond between steel & concrete	80	lbs.

REINFORCED CONCRETE FLOOR SYSTEM

10 Tension in concrete	00	lbs.
Compression, class A concrete	700	"
Tension, reinforcing steel	16000	"
Bond between steel & concrete	80	"
Shear (Diagonal tension) in concrete when no shear reinforcement is provided	40	"
Shear (Diagonal Tension) in concrete when reinforcement is provided for shear in excess of 40 lbs. per sq. inch	120	"

REINFORCED CONCRETE RIBS

11 Tension in concrete	00	lbs.
Compression in concrete classA, due to dead and live loads	700	"
Compression in concrete class A, due to dead and live loads, temperature and rib shortening combined	800	"

Shear (Diagonal Tension) when no shear reinforcement is provided	40 Lbs.
Shear (Diagonal Tension) when reinforcement is provided for shear in excess of 40 lbs. per sq. inch.	120 "
Bond between reinforcing steel and concrete	80 "
Tension in reinforcing steel	16000 "

12 Temperature stresses induced by a range of 30 degrees either way from normal shall be provided for in addition to dead and live load stresses.

SPANDREL COLUMNS

13 Compression in concrete Class "A" in longitudinally reinforced column	400 Lbs.
Compression in concrete Class "A" for combination of longitudinal and spiral reinforcement	500 "

PIERS AND ABUTMENTS

Concrete Piers

Massive Concrete

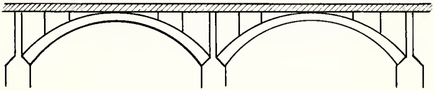
14 Massive concrete shall be used for piers that are liable to be submerged in water, in order to obtain the greatest possible weight.

Ice Pressure

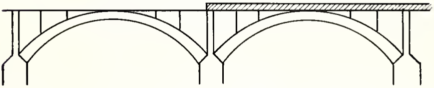
15 Ice pressure against concrete piers shall be taken as equal to the water line on the exposed side into the maximum depth of ice, multiplied by

900 lbs. per square foot.

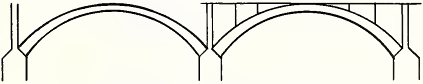
Impact 16 Concrete piers shall be proportioned to withstand impact from floating debris such as logs, etc. to which they are likely to be subjected.



D. L. Complete
Uniform L.L. over both spans



D.L. only Full D.L. and L.L.



D.L of ribs only Full D.L.

17 Combinations of Loadings for Piers

CONCRETE ABUTMENTS

18 U-abutments shall be used, constructed of massive concrete.

Overturning

19 Abutments shall be designed so as to be stable against overturning by revolving on the line formed by the intersection of the front face and the soil.

Crushing

20 Abutment shall be safe against crushing on the line formed by the intersection of the front face and the soil.

Sliding

21 Abutment shall be designed to resist sliding of abutment upon foundation bed.

Maximum Soil Pressure

22 All abutments shall be founded on rock, a maximum allowable pressure of 18 tons per square foot at the toe.

Drainage

23 In the design of the abutment, provision shall be made for adequate drainage.

Waterproofing

24 The inside surfaces of retaining walls shall be waterproofed with a heavy, impervious coating of hot waterproofing pitch, with two-ply elastic fabric, 2 ft. in width, over all expansion joints. The elastic fabric shall be "Tartar" or its equal, applied by first swabbing the concrete with hot pitch and then applying two alternate layers of fabric and pitch.

EXPANSION JOINTS

25 Expansion joints in roadway and sidewalk slab shall be provided for every 25 feet throughout the structure.

U - shaped copper plates filled with asphalt shall be used in roadway slabs to close the opening. Similar plates placed in the reversed position to those in the roadway slabs and with the asphalt filling omitted, shall be used in the sidewalk slabs and curbs.

26 Expansion joints for stringers shall be provided for as shown on plans. All expansion joints shall be packed with tar paper and asphalt to form a closed but compressible joint. Planed steel bed and bearing plates shall be used under the expansion end of each stringer. These plates shall have anchorage angles riveted to them and are to be further anchored with bent anchor bolts passing thru holes in these angles.

Sliding members and those on which the sliding members rest are to be reinforced at these points with loops of steel to provide for shearing stresses, which result from the friction of the plates on each other. In order to reduce this friction as much as practicable the bed and bearing plates shall be planed in the direction of movement, and the sliding surfaces in contact shall be heavily coated with tallow.

The stringers shall slide in recesses or pockets in floor beams.

SHRINKAGE REINFORCEMENT

27 In addition to the steel reinforcement required to resist the calculated stresses in the members, a certain amount of steel shall be used to prevent the formation of shrinkage settlement and temperature cracks. This steel shall be used in the exterior faces of walls, in a longitudinal direction in the slabs, and to reinforce the corners where two walls join. All braces and ribs shall be wrapped with 1/4" diameter wire spaced 6 inches, to which longitudinal steel is secured.

MATERIALS - GENERAL

Concrete Work

28 Three classes of concrete will be used in the bridge. Class "A" consists of 1 : 2-1/2 : 5 concrete with embedded stones, the stones to be placed not nearer than 6 inches to each other, nor nearer than 9 inches to any exposed face.

This concrete is to be used in abutments and piers. Class "B" consists of 1 : 2 : 4 concrete, and it is to be used in arch ribs, retaining walls, spandrel columns, horizontal braces and floor system.

Cast stone shall be used for ornamental work, railings and posts. In the construction of the railing, the base, the cap and all posts, the concrete shall be cast in place and given a rubbed finish except when bush-hammered surfaces are specified. The webs shall be cast in

units and removed from the molds while green, then scrubbed with brushes to remove the skin from same and expose the aggregate. All cast stone shall be made of the proportions of 1 part Portland cement, 1 to 1-1/2 parts Canadian sand, 1 part limestone (1/4 to 1/2 inch in size), and 1 to 1-1/2 parts silica gravel (1/4 to 3/8 inch in size)

The engineer is to determine the exact proportions from sample blocks to be made by the contractor at his expense.

All concrete proportions are based on the assumption that 1 bbl. of Portland cement is equivalent to 3.8 cubic feet. The quantities of materials required for 1 cu. yard of concrete of various proportions are given in the accompanying table:

Proportion	Cement Bbls	Sand Cu.Yds	Stone or gravel Cu.Yds
1 : 1-1/2 : 3	2.00	0.42	0.84
1 : 2 : 4	1.57	0.44	0.88
1 : 2-1/2 : 5	1.30	0.46	0.92

Reinforcing Steel

29 Vertical stirrups shall always pass under the main tension bars and be securely wired thereto. All reinforcing bars shall be medium open hearth steel, and shall be wired together at every intersection. Splices other than those in the arch ribs shall at least be forty diameters in length and shall be securely wired in at least two places.

Each arch rib splice shall have three special U-bolt clamps, as

shown on plans.

Forms and Centers

30 Forms shall in general be composed of 2 inch matched sheeting and shall be surfaced on the side next to the concrete for all surfaces which will be exposed in the finished work. In general, forms shall not be removed from concrete in less than 14 days in summer and 28 days in winter, except for vertical surfaces of members which carry no loads, the forms of which may be removed in from 3 to 6 days. Centers shall not be struck from arches in less than 28 days, and in no case without first notifying the engineer in charge.

In proportioning forms and centering, the concrete shall be treated as a liquid of its full weight for vertical loads and one-half its weight for horizontal pressures. In constructing timber centers for arches, an allowance for settlement shall be made equal to 3/4 inch for each 30 feet of vertical height.

Surface Finish of Concrete

31 The exposed surfaces of arches, piers, spandrel columns, railings, beams, retaining walls and abutments shall be finished by rubbing in a thin mortar with carborundum brick, sufficient to fill all pores and cavities and to remove all board marks. Where indicated on the drawings, the panels shall be bush-hammered.

Pavement

32 The paving of the bridge shall consist of 12 inches of macadam, the top surface of which is bound by

Standard Asphalt macadam binder, applied hot at the rate of about 2-1/2 gallons per square yard.

FORMULAS USED IN DESIGN

- Notation -

1. Rectangular Beams

f_s = tensile unit stress in steel.

f_c = compressive unit stress in concrete.

E_s = modulus of elasticity of steel.

E_c = modulus of elasticity of concrete.

$$n = \frac{E_s}{E_c}$$

M = bending moment.

A = steel area.

b = breadth of beam.

d = depth of beam to center of steel.

k = ratio of depth of neutral axis to effective depth d.

j = ratio of lever arm resisting couple to depth d.

jd = arm of resisting couple.

p = steel ratio. (Not percentage)

R = coefficient of strength.

2. T - Beams

b = width of flange.

b' = width of stem.

t = thickness of stem.

3. Beams Reinforced for Compression

A' = area of compressive steel.

p' = steel ratio for compressive steel.

f'_s = compressive unit stress in steel.

d' = depth to center of compressive steel.

4. Shear and Bond.

V = total shear.

v = shearing unit stress.

u = bond stress per unit area of bar.

o = perimeter of bar.

Σ_o = sum of the perimeters of all bars.

5. Columns

A = total net area.

A_s = area of longitudinal steel.

A_c = area of concrete.

P = total safe load.

p_s = steel ratio (hoops)

p = steel ratio (longitudinal)

d = diameter of hoop wire.

i = pitch of hoop in inches.

D = effective diameter of column, in inches.

6. Flexure and Direct Stress

a = distance from steel to center of section for symmeterical reinforcement.

f_c = maximum compressive fibre stress in concrete.

f_s = stress in steel near tension face.

A_t = area of transformed section.

e = eccentric distance.

$\frac{e}{h}$ = eccentricity.

FORMULAS

1. Rectangular Beams.

(1) $k = \sqrt{2pn + (pn)^2} - pn$

(2) $M = R \cdot bd^2$ $\qquad$ $f_s = \dfrac{M}{jdA}$ (4)

(3) $p = A/bd$ $\qquad$ $f_c = \dfrac{2M}{jdbkd}$ (5)

2. T - Beams

Case I Neutral Axis in Flange.

Formulas (1) to (5) as for rectangular beams, Formula (1) for k will determine whether the case is I or II

Case II Neutral Axis in the Web.

(6) $k = \dfrac{nA + .5bt \times t/d}{nA + bt} = \dfrac{pn + .5\,(t/d)^2}{pn + t/d}$

(7) $z = \dfrac{3kd - 2t}{2kd - t} \cdot \dfrac{t}{3}$

(8) $f_s = \dfrac{M}{Ajd}$

(9) $f_c = \dfrac{Mkd}{bt(kd - .5t)jd} = \dfrac{f_s}{n} \cdot \dfrac{k}{1-k}$

3. Beams Reinforced for Compression

(10) $k = \sqrt{2n(p + p'd'/d) + \left[n(p + p')\right]^2} - n(p + p')$

(11) $j' = (1 - \frac{z}{d})$

(12) $f_s = \frac{M}{j'dA}$ (13) $f_c = \frac{k}{n(1 - k)f_s}$

4. Shear, Bond and Web Reinforcement

(14) $v = \frac{V}{bjd}$

(15) $u = \frac{V}{jd\Sigma_o}$

Vertical Web Reinforcement

(16) $P = Af_s = \frac{(V - V_c)s}{jd} = (V - V_c)b's$

Web Reinforcement inclined at 45 degrees.

(Bent up Bars)

(17) $P = Af_s = 0.7 \frac{(V - V_c)s}{jd} = .7(V - V_c)b's$

(18) $V_c = 40bd$

(18a) $v_c = 40\#$ per sq. inch.

5. Columns.

Longitudinally reinforced columns

(19) $P = 400A(1 + 14p)$

Combined longitudinal and spiral reinforcement in columns.

(20) $P = 500A(1+14p)(1+35p_s)$

(21) $p_s = \frac{\pi d^2}{iD}$

(22) $i = \frac{D}{10}$

6. Flexure and Direct Stress

Case I. Compression on entire cross-section.

Symmetrical reinforcement.

$$\frac{12k(1+2np)e}{h} = 1 + \frac{24npa^2}{h^2} + \frac{6(1+2np)e}{h} \qquad (24)$$

(25) $\frac{M}{bh^2f_c} = \frac{1}{12k}(1 \quad 24np^2/h^2)$

(26) $f_s = nf_c$

Case II. Tension on part of the cross-section.

Tension in concrete neglected.

Symmetrical reinforcement.

(27) $k^3 - 3(.5-\frac{e}{h})k^2 + 12np\frac{e}{h}k = 6np(\frac{e}{h} + \frac{2a^2}{h^2})$

(28) $\frac{M}{bh^2f_c} = \frac{1}{12}k(3-2k) + \frac{2pn}{k} \cdot \frac{a^2}{h^2}$

(29) $f_s = nf_c$

Chapter II

DESIGN OF A REINFORCED CONCRETE ARCH HIGHWAY BRIDGE

Art. 1 General

The bridge has a total length of 317 feet 6 inches out to out and a 24 foot clear roadway. It has two 3 ft. 9 inch sidewalks and is provided with an ornamental concrete railing which extends 3 ft. 9 inches above the sidewalk. surface.

Starting at the east end of the bridge, the composition of the structure is as follows: The U-abutment has 20 feet of retaining wall construction, on which rest the floor system. The deck was continued over the abutment so that settlements of the fill will not produce failures in the sidewalks or pavement. Each of the arches has a clear span of 85 feet and a rise of 14 feet 3 inches, and consists of two fixed arch ribs, carrying spandrel columns which support the floor system. The arch ribs are spaced 24 feet center to center and are uniformly 2 feet

6 inches wide and increase in depth from 3 feet 8 inches at the crown to 6 feet 2 inches at the springing line. The arch ribs are braced together at the foot of each spandrel post by transverse braces which are of the same width as the spandrel posts and floor beams, and are as deep as the ribs will permit. These braces a re placed in vertical planes.

The piers of each rib are independent save for struts between them. The piers and abutments are founded on rock. Both abutments are of similar construction.

Provision is made for carrying electric wires and small pipes under the sidewalks by means of ducts leading into the railing posts and up to the lighting fixtures.

Design of Floor System.

In the design of the floor system, plates III, VI, X, and XII in the work " Principles of Reinforced Concrete Construction" by Turneaure and Maurer were used.

Art. 2 Design of Roadway Slab.

D. L. per foot of slab, macadam = 140#

D. L. " " " " (assumed) = 94

Total D. L. per foot of slab 234#

D. L. Bending Moment = $\frac{234 \times 6 \times 12}{10}$ = 10100" #

L. L. B. M. (concentrated load) =

.170 x 14000 x 6 x 12 = 63544" #

Total Bending Moment = 250004" #

D. L. Shear .6 x 234 x 3 = 422#

L. L. Shear .6 x 14000 = 8400

Impact, 40% = 3600

Total Shear = 12182#

f_c = 700# /sq.in. f_s = 16000# /sq.in.

R = 120 p = .0087

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{250004}{46 \times 120}} = 6.72''$$

Use $d = 6.75''$ $h = 7.5''$

$$v = \frac{V}{bd} = \frac{12182}{46 \times 6.75} = 39\# \text{ per sq. inch}$$

$A = .0087 \times 6.75 \times 12 = .705$ sq. inches

5/8" bars 5" c. to c.

$$u = \frac{12182}{(46/5)(2.5)(.87)(6.75)} = 81\# \text{ per sq. in.}$$

$$\text{Wt. of slab} = \frac{7.5 \times 12 \times 150}{144} = 93.8 \quad \text{(ok)}$$

3/8" bars placed 12" c. to c.

(Temperature and shrinkage)

Art. 3 Design of Stringer S_1

D. L. per ft. of stringer (macadam) = 6 x 140 = 840#

D. L. " " " " slab = 6 x 94 = 564

Stringer (aaumed) 190

Total 1594#

D. L. Shear = 5.17 x 1594 = 8250#

L. L. Shear = .150 x 14000 = 7000#

Impact " = 40% = 2800#

Total Shear = 18050#

$$b'd = \frac{18050}{105} = 172'' \quad b' = 9'' \quad d = 19.5''$$

$$h = 21''$$

$$\text{Wt. stem} = \frac{9 \times 21 \times 150}{144} = 197\# \quad \text{(ok)}$$

$$\text{D. L. B. M.} = \frac{1594 \times 10.33^2 \times 12}{12} = 170000$$

$$\text{L. L. B. M.} = .167 \times 14000 \times 10.33 \times 12 = 290000$$

$$\text{Impact} = 40\% \qquad = \underline{116000}$$

$$\text{Total} \qquad = 576000$$

$$A = \frac{M}{f_s jd} = \frac{576000}{16000 \times .88 \times 19.5} = 2.1 \text{ sq. in.}$$

$$p = \frac{2.1}{31 \times 19.5} = .0035 \qquad k = .27 \quad j = .91$$

$$A = \frac{576000}{16000 \times .91 \times 19.5} = 2.03 \text{ sq. inches}$$

8 - 5/8" dia. = 2.454 sq. in.

Bond stress along the top of stringer near support (assume 8 - 5/8" rods)

$$u = \frac{18050}{8(1.96)(.85)(19.5)} = 70\# \text{ per sq. in.}$$

$$\text{At support} \quad p' = \frac{2.454}{9 \times 19.5} = .014$$

$$k = .50 \qquad j = .84$$

$$f_c = \frac{2M}{kjbd^2} = \frac{2 \times 576000}{.50 \times .84 \times 9 \times (19.5)^2} = 800\#$$

$$\frac{f_c - f'_c}{f_c} = \frac{800 - 750}{800} = 6.25\%$$

Plate XII $p' = .002$ (req'd for compression)

Actual $p' = .014$ (ok)

Web Reinforcement

Stress to be taken by 4 - 5/8" dia. rods is equal to $P = .7(V - V_c)b's =$

$$.7 \times \frac{40 + 60}{2} \times 9 \times 14.6 = 4600\#$$

$$f_s = \frac{.7(V - V_c)b's}{A} = \frac{4600}{2.454} = 1880\# \text{ per sq.in.}$$

Diagonal tension near center and at ends will be cared for by addition of stirrups.

$$i = 2.4 \frac{u}{f_s} d = .012d = .235"$$

1/4" dia. stirrups used.

Spacing at support.

$$V = 18050$$

$$V_c = 35 \times 9 \times 19.5 = \underline{6140}$$

$$V - V_c = 11910\#$$

$$S = \frac{Af_s jd}{V - V_c} = \frac{.098 \times 16000 \times .88 \times 19.5}{11910}$$

$$S = 2"$$

$\frac{1}{8} = 1.29$ $s = 4/3(2) = 3$

$\frac{1}{4} = 2.58$ $s = 2(2) = 4"$

Art. 4 Design of Sidewalk Slab - Span 3' 0"

D. L. per foot slab (assumed) = 38#

L. L. = 100

Total = 138#

$$\text{D. L. B. M.} = \frac{38 \times 3 \times 3 \times 12}{18} = 514$$

$$\text{L. L. B. M.} = \frac{100 \times 3 \times 3 \times 12}{18} = 1350$$

Impact = 40% = 540

Total = 2404#

$$d = \sqrt{\frac{2404}{12 \times 120}} = 1.3" \quad \text{Use } d = 2" \quad h = 2.5"$$

$$V = \frac{178 \times 1.5}{12 \times 2} = 15\# \text{ per sq. in.}$$

$A = .0087 \times 2 \times 12 = .207$

3/8" dia. bars 8" c. to c.

$$u = \frac{1.78 \times 1.5}{12/8\,(1.50)(.81)(2)} = 72\# \text{ per sq.in.}$$

3/8" dia. placed 18" c. to c. (Temperature bars)

Art. 5 Design of Stringer S_2

D. L. Roadway slab, wt. per ft.		= 282#
D. L. Sidewalk " " " "	1.5 x 38	= 57
L. L. on roadway 125 x 3		= 375
L. L. on sidewalk 100 x 1.5		= 150
D. L. Stringer 21.5 x 12		= 270
		1134#

$$\text{D. L. B. M.} = \frac{609 \times 10.3^2 \times 12}{10} = 78000 \text{ in.lbs.}$$

$$\text{L. L. B. M.} = \frac{525 \times 10.3^2 \times 12}{10} = 67200 \text{ " "}$$

Impact, 40% = 26880 " "

172080 " "

D. L. Shear = 5.17 x 609 = 3150

L. L. Shear = 5.17 x 525 = 2665

Impact, 40% = 1066

6881#

$$d = \sqrt{\frac{172080}{10 \times 120}} = 12"$$

$$bd = \frac{6881}{35} = 198 \text{ sq. in.} \qquad b = 10" \quad d = 20"$$

$h = 21.5"$

A = .0087 x 10 x 20 = 1.74 sq. in.

4 - 3/4" dia. = 1.767 sq. in.

Web reinforcement, 2 - 3/4" dia. will be bent up and 3 stirrups will be spaced 12" & 1 - 18" 1/4" dia. stirrups.

Art. 6 Stringer S_3

D. L. per ft. stringer (s.w.slab) 38 x 1.5 = 57
D. L. " " " (assumed) 8 x 14 = 117
D. L. " " railing 3.5 x 150 = 525
L. L. " " stringer 100 x (1.5-1) = 50
749

D. L. Shear = 699 x 5.17 = 3615
L. L. " = 50 x 5.17 = 259
Impact, 40% = 104
3978#

$bd = \frac{3978}{35} = 113$ sq. in. $b = 9"$ $d = 12.5"$
$h = 14"$

$a_s = .0087 \times 9 \times 12.5 = 0.97$ sq. in.

4 - 5/8" dia. = 1.227 sq. in.

$$u = \frac{3978}{4 \times 1.96 \times .87 \times 12.5} = 48\#\text{ per sq.in.}$$

Web reinforcement - 2 - 5/8" dia. bent up and 3 stirrups 1/4" dia., 12" on centers.

Art. 7 Design of Beam B_1

Span 24ft. 0 in. Loads $\frac{S1}{2}$, S_2 , S_3

Concentrated load $\frac{S1}{2}$ (at $\frac{1}{2}$) = 18050#

" " S_1 (at $\frac{1}{3}$) = 36100

" " $\frac{S_2}{2}$ (at 1) = 6881

Beam weight 760 x 12 = 9120

Total Shear = 70151#

Max. B. M. = (7015 x 12 - 6881 x 12 - 36100 x 6) 12

- 360000 = 6,151,680 in. lbs.

$b'd = \frac{70151}{105} = 669$ $b' = 19$ $d = 35.5"$ $h = 37.5"$

wt. stem = $\frac{19 \times 37.5 \times 150}{144} = 750$ # (ok)

$\frac{M}{bd^2} = \frac{6151680}{79 \times (35.5)^2} = 62$ $\frac{t}{d} = \frac{7.5}{35.5} = .21$

f_c = 500# per sq. in. $j = .91$

$A = \frac{6151680}{16000 \times .91 \times 35.5}$ = 11.93 sq. in.

12 - 1 1,8" dia. = 11.928 sq. in.

Bond stress along top of beam near support

6 - 1 1/8" dia. and 4 - 5/8" dia.

$$u = \frac{70151}{(6 \times 3.53 + 4 \times 1.96)(.87)(35.5)} = 78\#/\text{sq.in.}$$

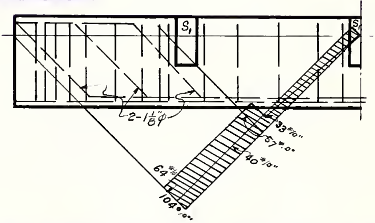
$k = .53$

At support $p' = \frac{13.155}{35.5 \times 19} = .0195$

$j = .82$

$$f_c = \frac{2 \times 306000}{.53 \times .82 \times (35.5)^2(19)} = 60\#/\text{sq.in.}$$

Web Reinforcement



Stress to be taken by 6 - 1 1/8" dia. is equal to

$$P = .7(V-V_c)b's = .7\left(\frac{64+57}{2}\right)19 \times 24 = 19500\#$$

$$f_s = \frac{.7(V-V_c)b's}{A} = \frac{19500}{5.96} = 3270\#/\text{sq.in.}$$

Diagonal tension between stringers and at end of beam will be cared for by addition of stirrups.

$$i = \left(2.4 \frac{u}{f_s}\right)d = .012 \times 35.5 = .48''$$

1/2" dia. stirrups are used.

Spacing at support.

$V = 70151\#$

$V_c = 35 \times 19 \times 35.5 = 23600$

$V - V_c = 46551\#$

$$s = \frac{Af_s jd}{(V-V_c)} = \frac{.396 \times 16000 \times .88 \times 35.5}{46551}$$

$s = 4''$ $s_1 = 6''$ $s_2 = 12''$

Art. 8 Design of B_2 (Cantilever)

Span 3' 0"

Concentrated load S_2 (at end) $= 13762\#$

" " S_3 (at end) $= 7956$

Wt. of Beam $(\frac{12+35}{2})36 \times 19 \times \frac{150}{12 \times 12 \times 12} = \underline{1300}$

Total shear at end (inner) 23018#

Shear taken by concrete at the inner end =

$35 \times 19 \times 35 = 23275\#$

No web reinforcement needed.

Actual shear at $\frac{1}{2}$ = 8590#

Concrete can take $24 \times 19 \times 35 = 15950\#$

No web reinforcement needed.

Web reinforcement, 3 - 3/8" dia. stirrups spaced 6" will be used.

Max. B. M. = 7956 x 36 + 1300 x 15 = 306000" #

$$A = \frac{306000}{16000 \times .88 \times 35} = 0.62 \text{ sq. in.}$$

4 - 5,8" dia. are used.

Art. 9 Design of Beam B_3

Span 33' 5" Loads S_2 , S_1 , and $\frac{S1}{2}$

Concentrated load S_2 = 13762

" " S_1 = 36100

" " $S_1/2$ = 18050

Beam weight = 14000

Total Shear= 81912#

Max. B. M. = 81912 x 16.71 - 13762 x 12 -

36100 x 6 - 835 x 16.71 x 8.35 = 871656' #

= 10,459,872" #

$b'd = \frac{81912}{105} = 780$ sq.in. b' = 21 d = 37"

h = 39"

Wt. of Stem = $\frac{21 \times 39 \times 150}{144}$ = 835#/' (ok)

$\frac{M}{bd^2} = \frac{10,459,872}{81 \times (37)^2} = 94$ $\frac{t}{d} = \frac{7.5}{37} = .20$

f_c = 675#/sq.in. j = .91

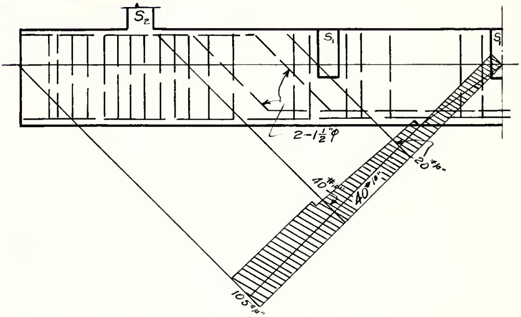
$$A = \frac{10459872}{16000 \times .91 \times 37} = 19.40 \text{ sq. in.}$$

11 - 1 1,2" dia. = 19.44 sq. in.

Bond Stress

$$u = \frac{81912}{11 \times 4.7 \times .87 \times 37} = 50\#/\text{sq.in.}$$

Web Reinforcement.



Stress to be taken by 4 - 1 1/4" dia. is equal

$$\text{to } P = .7\left(\frac{40+20}{2}\right)21 \times 29 = 12800\#$$

$$f_s = \frac{12800}{4.91} = 2600\#/\text{sq.in.}$$

Diagonal tention at the ends will be cared for by addition of stirrups.

$i = .012 \times 37 = .45"$

1/2" dia. stirrups are used.

Spacing of stirrups

$V = \qquad\qquad = 81912$

$V_c = 37 \times 21 \times 35 = \underline{27200}$

$V - V_c \qquad = 54712\#$

$S = \dfrac{.392 \times 16000 \times .87 \times 37}{54712} = 4''$ (at end)

$S_1 = 6'' \qquad S_2 = 9''$

Art. 10 Design of Beam B_4

Span 33' 5" Loads $S_2/2$, $S_1/2$ and $S_1/4$

Concentrated Load $S_2/2$ = 6881#

" " $S_1/2$ = 18050

" " $S_1/4$ = 9025

Beam Weight = $\underline{8000}$

Total Shear = 41956#

$\text{Max. B. M.} = 41956 \times 16.71 - 6881 \times 12 - 18050 \times 6$

$- 442 \times 16.71 \times 8.35 = 449528'\# = 5394335''\#$

$b'd = \dfrac{41956}{105} = 400$ sq. in. $\quad b' = 14 \quad d = 28.5''$

$h = 30.5''$

$\text{Wt. stem} = \dfrac{14 \times 30.5 \times 150}{144} = 442\#/'$ (ok)

$\dfrac{M}{bd^2} = \dfrac{5394336}{81 \times (28.5)^2} = 82 \qquad \dfrac{t}{d} = \dfrac{7.5}{28.5} = .26$

f_c = 550#/sq.in. j = .89

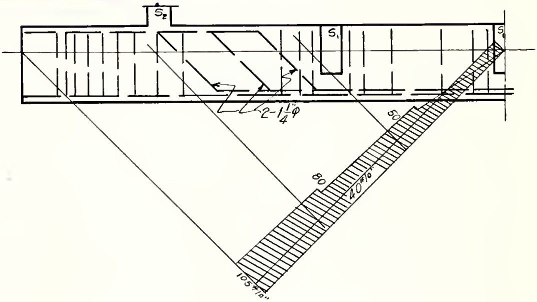
$$A = \frac{5394336}{16000 \times .89 \times 28.5} = 13.30 \text{ sq.in.}$$

11 - 1 1/4" dia. = 13.50 sq. in.

Bond Stress -

$$u = \frac{41956}{11 \times 3.9 \times .87 \times 28.5} = 40\#/\text{sq.in.}$$

Web Reinforcement.



Stress to be taken by 6 -1 1/4" dia. is equal to

$$P = .7\left(\frac{60 + 30}{2}\right)14 \times 20 = 8830\#$$

$$f_s = \frac{8830}{7.36} = 1200\#/\text{sq.in.}$$

Diagonal tension at the ends of beams and between stringers will be cared for by addition of stirrups.

$i = .012 \times 28.5 = .34$

3, 8" dia. stirrups are used.

Spacing of stirrups.

$V = 41956\#$

$V_c = 35 \times 28.5 \times 14 = \underline{13970}$

$V - V_c = 27986$

$$S = \frac{.221 \times 16000 \times .87 \times 28.5}{27986} = 3''$$

$S_1 = 6''$ $S_2 = 9''$

DESIGN OF SPANDREL COLUMNS

Art. 11 Design of Column C_1

Length = 4.0

Loading on column C_1 = 108669#

Wt. of Col. C_1, 19 x 19 = $\underline{570}$

Total Load 109239#

Longitudinal reinforcement only

$P = f_c A(1 + 14p)$ $A = 15'' \times 15''$

$$(1 + 14p) = \frac{P}{f_c A} = \frac{109899}{400 \times 225} = 1.21$$

$(1 + 14p) = 1.21 \quad p = .015$

$a_s = .015 \times 225 = 3.38$ sq. in.

8 - 3/4" dia. = 3.54 sq. in.

1/4" dia. wire pitch 6"

Wt. col. = $\frac{19 \times 19 \times 150}{144} \times 1.5 = 570\#$ (ok)

Column Section 19" x 19"

Art. 12 Design of Column C_2

Length = 7.3'

Load on column C_2 = 108669#

Wt. of Col. 19 x 22 = 2400

111069#

$P = 500A(1 + 14p)(1 + 35p_s)$

$(1 + 35p_s) = \frac{111069}{500 \times 270 \times 1.28} = .642$

$p_s = .0103$

$d^2 = \frac{1.0103 \times 2 \times 18}{3.14} = .325"$

d = .57" Use 5/8" dia. hoops i = 2"

p = .02 x 270 = 5.40 9 - 7/8" dia. = 5.41 sq.in.

Column section 19" x 22"

Wt. column = $\left(\frac{19 \times 22 \times 150}{144}\right) 5 = 2200\#$ (ok)

Art. 13 Design of Column C3

Length 12' 0"

Load on column C_3		=	108669#
Wt. Column C_3	19 x 24	=	4800
			113469#

$$(1 + 35p_s) = \frac{113469}{500 \times 15 \times 20 \times 1.28} = .591$$

$p_s = .0117$

$$d^2 = \frac{1.0117 \times 2 \times 20}{3.14} = .149$$

d = .387" Use 1,2" dia. hoops $i = 2"$

p = .02 x 300 = 6.00 sq. in.

10 - 7,8" dia. = 6.013 sq. in.

$$\text{Wt. Col.} = \frac{19 \times 24 \times 150}{144} \times 9.5 = 4520 \quad \text{(ok)}$$

Column section 19" x 24"

Design of Arch Ribs

Art. 14 General

Span 85' 0" Rise 14' 3"

Section of arch at crown 44" x 30"

Section of arch at springing line 74" x 30"

$$\text{Intrados radius} = \frac{(.5\ \text{Span})^2 + (\text{Rise})^2}{2 \times \text{Rise}} =$$

$$\frac{42.5^2 + 14.25^2}{2 \times 14.25} = 70.50\ \text{Ft.}$$

$$\text{Axis Radius} = \frac{44.33^2 + 13.58^2}{2 \times 13.58} = 78.91\ \text{Ft.}$$

$$\text{Extrados Radius} = \frac{46.25^2 + 12.95^2}{2 \times 12.95} = 89.06\ \text{Ft.}$$

Reinforcement of each arch rib is :

14 - 1 1/8" dia. rods and stirrups placed as shown on plans.

Art. 15 Arch Loading

Dead Load of Deck

Macadam = 10.33 x 12 x 140 =	17360#
Roadway slab = 10.33 x 12 x 94 =	11650
Sidewalk = 10.33 x 4 x 38 =	1570
Stringer S_1 = 1.5 x 10.33 x 197 =	3070
" S_2 = 10.33 x 270 =	2790
" S_3 and railing = 642 x 10.33 =	6630
Beam B_1 = 12 x 750	9000
" B_2 and coping = 1400 =	1400
Total D. L. on column =	53470#

Live Load on Deck

L. L. on sodewalk = 10.33 x 4 x 100 =	4132#
L. L. on roadway = 10.33 x 12 x 125 =	15500
	19632
Impact 40% =	7853
Total L. L. on column =	27485#

Sec. 15 Arch Loading

Loads	1	2	3	4	5
Deck	26735	53470	53470	53470	53470
Column			570	2200	4520
Arch	13200	13863	15320	17240	20290
D. L.	39935	67333	69360	72910	78280
L. L.	13743	27485	27485	27485	27485
D.+L.Load	53678	94818	96845	100395	105765

TABLE NO. 1

Divisions of Arch Rib

Preliminary Divisions					Final Divisions		
No. of Division	Depth d	I_c	$14nI_s$	$I = I_c + 14nI_s$	ds	I	$\frac{ds}{I}$
C	3.67	10.29	3.26	13.55			
1	3.71	10.62	3.35	13.97	3.10	13.97	.22
2	3.75	10.96	3.44	14.40	3.15	14.40	.22
3	3.84	11.79	2.68	14.47	3.25	14.50	.22
4	4.00	13.31	4.04	17.35	3.25	14.50	.22
5	4.21	15.53	4.54	20.07	3.60	16.00	.22
6	4.46	18.45	5.18	23.63	4.00	18.40	.22
7	4.75	22.30	6.03	28.33	4.60	21.00	.22
8	5.00	26.00	7.14	33.14	5.50	25.00	.22
9	5.50	34.60	8.49	43.09	6.75	31.80	.21
10	5.96	44.01	10.18	54.19	10.05	48.00	.21
S	6.17	48.81	10.96	59.77			

TABLE NO. 2

Pt.	Dead Load Bending Moments	Live Load Bending Moments
1	39935 x 1.58 = 63097.30	13743 x 1.58 = 21713.94
2	39935 x 4.67 = 186496.45	13743 x 4.67 = 64179.81
3	39935 x 7.83 = 312691.05	13743 x 7.83 = 107607.69
4	39935 x 11.08 = 442479.80	13743 x 11.08 = 152272.44
	67333 x 0.75 = 50499.75	27485 x 0.75 = 20613.75
5	39935 x 14.67 = 585846.45	13743 x 14.67 = 201609.81
	67333 x 4.34 = 292225.22	27485 x 4.34 = 119284.90
6	39935 x 18.50 = 738797.50	13743 x 18.50 = 254245.50
	67333 x 8.17 = 550110.61	27485 x 8.17 = 224552.45
7	39935 x 22.75 = 908521.25	13743 x 22.75 = 312653.25
	67333 x 12.42 = 836275.86	27485 x 12.42 = 341361.70
	67360 x 2.08 = 144268.80	27485 x 2.08 = 57168.80
8	39935 x 27.54 = 1099809.90	13743 x 27.54 = 378482.22
	67333 x 17.21 = 1158800.93	27485 x 17.21 = 473016.85
	69360 x 6.88 = 477196.80	27485 x 6.88 = 189096.80
9	39935 x 33.21 = 1326341.35	13743 x 33.21 = 456405.03
	67333 x 22.88 = 1540579.04	27485 x 22.88 = 628856.80
	69360 x 12.55 = 870468.00	27485 x 12.55 = 344936.75
	72910 x 2.22 = 161860.20	27485 x 2.22 = 61016.70
10	39935 x 40.38 = 1612575.30	13743 x 40.38 = 554942.34
	67333 x 30.05 = 2023356.65	27485 x 30.05 = 825924.25
	69360 x 19.72 = 1367779.20	27485 x 19.72 = 542004.20
	72910 x 9.39 = 684624.90	27485 x 9.39 = 258084.15
S	39935 x 44.33 = 1770318.55	13743 x 44.33 = 609227.19
	67333 x 34.00 = 228932.00	27485 x 34.00 = 934490.00
	69360 x 23.67 = 1641751.20	27485 x 23.67 = 650569.95
	72910 x 13.34 = 972619.40	27485 x 13.34 = 366649.90
	78280 x 3.01 = 235622.80	27485 x 3.01 = 82729.85

TABLE Nº 3

CALCULATIONS FOR H_o, V_o AND M_o LIVE LOAD OVER $\frac{1}{2}$ SPAN

PT.	X	y	X^2	y^2	m_L	m_R	$(m_L+m_R)y$	$(m_R-m_L)X$
1	1.58	0.04	2.50	0.00	-84811	-63097	-5916	34308
2	4.67	0.17	21.81	0.03	-250676	-186497	-74319	299716
3	7.83	0.42	61.31	0.18	-420299	-312691	-307856	842571
4	11.08	0.79	122.77	0.62	-665866	-492980	-915587	1915577
5	14.67	1.38	215.21	1.91	-1198966	-878072	-2866312	4707515
6	18.50	2.17	342.25	4.71	-1767706	-1288908	-6632852	8857763
7	22.75	3.38	517.56	11.42	-2600250	-1889066	-15173898	16179436
8	27.54	5.00	758.45	25.00	-3776404	-2735808	-32561060	28658004
9	33.21	7.31	1102.90	53.44	-5390365	-3899149	-67906348	49523283
10	40.38	11.17	1630.55	124.77	-7869291	-5688336	-151438694	88066963
Σ		31.83	4775.31	222.08	-24024634	-17434604	-277882842	199085136
S	44.33	13.58			-9553301	-6909634		

$$H_o = \frac{n\Sigma my - \Sigma m \Sigma y}{2[(\Sigma y)^2 - n\Sigma y^2]} = \frac{10(-277882842) - (-41459238)(31.83)}{2[(31.83)^2 - 10(222.08)]} = +604216^{\#}$$

$$V_o = \frac{\Sigma(m_R - m_L)X}{2\Sigma X^2} = \frac{199085136}{2(4775.31)} = +20846^{\#}$$

$$M_o = -\frac{\Sigma m + 2H_o\Sigma y}{2n} = -\frac{(-41459238) + 2(31.83)(604216)}{2 \times 10} = +149742'^{\#}$$

TABLE Nº 4

BENDING MOMENTS. THRUSTS AND ECCENTRIC DISTANCES L.L. OVER $\frac{1}{2}$ SPAN

Pt.	$H_o y$	$V_o x$	BENDING MOMENT M		THRUSTS		ECCENTRIC DISTANCES	
			LEFT	RIGHT	LEFT	RIGHT	LEFT	RIGHT
1	24169	32937	+122037	+77877	604200	605000	0.20	0.13
2	102717	97351	+99134	-31389	605600	606000	0.16	-0.52
3	253771	163224	+146438	-72402	605600	607600	0.24	-0.12
4	477331	230974	+192181	-96881	618000	617000	0.31	-0.16
5	833818	305811	+90405	-200323	618000	618000	0.15	-0.32
6	1311149	385651	+78836	-213668	618000	618000	0.13	-0.35
7	2042350	474247	+66089	-171221	644000	636000	0.10	-0.27
8	3021080	574099	-31483	-139085	645000	636200	-0.05	-0.22
9	4416819	692296	-131508	-24884	686000	662000	-0.19	-0.04
10	6749093	841762	-128694	+368737	686800	663000	-0.19	+0.56
SP.	8205243	924103	-274213	+421248	742800	696000	-0.37	+0.61

TABLE Nº 5

CALCULATIONS FOR H_c, M_o, M, T, AND e. LIVE LOAD ON ENTIRE SPAN

PT.	$(m_L+m_R)y$	$H_o y$	BENDING MOMENT		THRUSTS		ECCENTRIC DISTANCE	
			LEFT	RIGHT	LEFT	RIGHT	LEFT	RIGHT
1	6785	27244	176962	176962	681100	681100	0.26	0.26
2	85230	115787	99640	99640	682000	682000	0.15	0.15
3	353051	286061	100291	100291	682000	682000	0.15	0.15
4	1052068	538067	106730	106730	696000	696000	0.15	0.15
5	3309146	939915	-24522	-24522	696000	696000	-0.04	-0.04
6	7671844	1477983	-55194	-55194	696000	696000	-0.08	-0.08
7	17577690	2302111	-63610	-63610	722400	722400	-0.09	-0.09
8	32764040	3405490	-136385	-136385	722400	722400	-0.19	-0.19
9	78807136	4978826	-177010	-177010	762000	762000	-0.23	-0.23
10	175799961	7607865	-26897	-26897	762000	762000	-0.35	-0.35
Σ	317426951							
SP		9249311	-69461	-69461	817000	817000	-0.85	-0.85

$$H_o = \frac{10(-317426951)-(-48049268)(31.83)}{2[(31.83)^2-10(222.08)]} = +681097^{\#}$$

$$M_o = -\frac{(-48049268)+2(681098)(31.83)}{2\times 10} = +234529^{\#}$$

TABLE No 6

CALCULATIONS FOR $H_o, M_o, M, T,$ AND e — DEAD LOAD ONLY

PT.	$(m_L+m_R)y$	H_oy	BENDING MOMENT		THRUSTS		ECCENTRIC DISTANCE	
			LEFT	RIGHT	LEFT	RIGHT	LEFT	RIGHT
1	5048	20242	89838	89838	506100	506100	0.18	0.18
2	63409	86029	32225	32225	507000	507000	0.06	0.06
3	262660	212542	32544	32544	507000	507000	0.06	0.06
4	778908	399782	39495	39495	516000	516000	0.08	0.08
5	2285479	698353	-47026	-47026	516000	516000	-0.09	-0.09
6	5593861	1098135	-58080	-58080	516000	516000	-0.11	-0.11
7	12770086	1710459	-45914	-45914	534800	534800	-0.09	-0.09
8	27358080	2530265	-72850	-72850	534800	534800	-0.14	-0.14
9	57005548	3699247	-67209	-67209	562000	562000	-0.12	-0.12
10	127077426	5652612	96969	96969	562000	562000	0.17	0.17
Σ	233200505							
SP		6872200	95259	95259	610000	610000	0.16	0.16

$$H_o = \frac{10(-233200505)-(-34869208)(31.83)}{2415} = +506053^{\#}$$

$$M_o = -\frac{(-34869208)+2(31.83)(506053)}{20} = +132693^{'\#}$$

CALCULATION OF STRESSES IN ARCH RIBS

Art. 16 Loading Stresses

Stresses at the crown sections numbers five, six and seven, and at the springing line shall be determined. The resultant forces due to the loadings all act within the middle third of the sections, therefore equations numbers (24), (25) and (26) page 20 shall be used in calculation.

Live load over one-half the span.

Stress at Crown.

$$\frac{12k(1+2np)e}{h} = 1+\frac{24npa^2}{h^2}+\frac{6(1+2np)e}{h}$$

$$k\ 1.064 = 2.164 \qquad k = 2.065$$

$$\frac{M}{bh^2 f_c} = \frac{1}{12k}(1 \quad 24npa^2/h^2)$$

$$f_c = \frac{149742}{2.5 \times 3.67^2 \times .066 \times 144} = 470\#/\text{sq.in.}$$

$$f_s = nf_c = 15 \times 470 = 7060\#/\text{sq.in.}$$

Stress at Section No. 5

$$12k(1+2 \times 5 \times .0098)\frac{.32}{3.95} = 1+\frac{24 \times 15 \times .0098 \times 1.64}{3.95}$$

$$\frac{6(1+2 \times 15 \times .0098)\ .32}{3.95} \qquad k = 1.78$$

$$f_c = \frac{200323}{2.5 \times 3.95^2 \times .075 \times 144} = 473\#/\text{sq.in.}$$

$$f_s = 15 \times 473 = 7100\#/\text{sq.in.}$$

Section at No. 6

$$12k(1+2\times15\times.0094)\frac{.35}{4.1} = 1+\frac{24\times15\times.0094\times1.72^2}{(4.1)^2}$$

$$+\frac{6(1+2\times15\times.0094)\ .35}{4.1} \qquad k = 1.72$$

$$f_c = \frac{213668}{2.5 \times 4.1^2 \times .077 \times 144} = 458\#/\text{sq.in.}$$

$$f_s = 15 \times 458 = 6900\#/\text{sq.in.}$$

Stress at Sections No. 7

$$12k(1+2\times15\times.0090)\ \frac{.27}{4.3} = 1+\frac{24 \times 15 \times .0090 \times 1.82^2}{4.3^2}$$

$$+\frac{6(1+2 \times 15 \times .0090)\ .27}{4.3} \qquad k = 2.16$$

$$f_c = \frac{171221}{2.5 \times 4.3^2 \times .061 \times 144} = 421\#/\text{sq.in.}$$

$$f_s = 15 \times 421 = 6320\#/\text{sq.in.}$$

Stress at Springing Line

$$12k(1+2\times15\times.0063)\frac{.605}{6.17} = 1+\frac{24\times15\times.0063\times5.50^2}{6.17^2}$$

$$k = 2.50$$

$$f_c = \frac{421248}{2.5 \times 6.17^2 \times .093 \times 144} = 329\#/\text{sq.in.}$$

$$f_s = 15 \times 329 = 5000\#/\text{sq.in.}$$

Live Load Over Entire Span

Stress at Crown

$$12k(1+2\times15\times.0105)\frac{.344}{3.67} = 1+\frac{24\times15\times.0105\times1.5^2}{3.67^2}$$

$$+\frac{6(1+2\times15\times.0105)\ .34}{3.67} \qquad k = 1.603$$

$$f_c = \frac{234529}{.085\times2.5\times3.67^2\times144} = 570\text{\#/sq.in.}$$

$$f_s = 15\times570 = 8550\text{\#/sq.in.}$$

Stress at Section No. 5

$$12k(1+2\times15\times.0098)\frac{.035}{3.95} = 1+\frac{24\times15\times.0098\times1.64^2}{3.95^2}$$

$$\frac{6(1+2\times15\times.0098\times.035}{3.95} \qquad k = 12.21$$

$$f_c = \frac{24522}{2.5\times3.95^2\times.01098\times144} = 400$$

$$f_s = 6000\text{\#/sq.in.}$$

Stress at Section No. 6

$$.296k = 1+.595+.148 \qquad k = 0.588$$

$$f_c = \frac{55194}{2.5\times4.1^2\times.226\times144} = 404\text{\#/sq.in.}$$

$$f_s = 15\times404 = 6060\text{\#/sq.in.}$$

Stress at Section No. 7

$$0.312k = 1+0.581+0.156 \qquad k = 5.56$$

$$f_c = \frac{63610}{2.5 \times 4.3^2 \times .0237 \times 144} = 403\#/\text{sq.in.}$$

$$f_s = 15 \times 403 = 6045\#/\text{sq.in.}$$

Stress at Springing Line

$$0.196k = 1 + 1.801 + 0.0982 \qquad k = 14.8$$

$$f_c = \frac{69461}{2.5 \times 6.17^2 \times .0158 \times 144} = 324\#/\text{sq.in.}$$

$$f_s = 15 \times 324 = 4860\#/\text{sq.in.}$$

Art. 17 Temperature Stresses

$$H_o = \frac{EI}{ds} \cdot \frac{ctln}{2\left[n\Sigma y^2 - (\Sigma y)^2\right]}$$

$$H_o = \frac{288000000}{.22} \cdot \frac{.000006 \times 30 \times 88.67 \times 10}{2(10 \times 222.08 - 31.83^2)}$$

$$H_o = 86400\#$$

$$M_o = -\frac{H_o \Sigma y}{n} = -\frac{86400 \times 31.83}{10} = -275012\#/\text{sq.in.}$$

Distance of equilibrium polygon below crown is equal to

$$\frac{M_o}{H_o} = \frac{-2705012}{86400} = -3.18'$$

Section No.	H_o	Lever Arm	Bending Moment
Crown	86400	3.18'	-275012'#
5	"	1.78	-153800'#
6	"	0.92	- 79488'#
7	"	0.20	- 17280'#
Spring'g L.	"	10.40	-898560'#

The thrust due to change in temperature acts outside the middle third at all sections except No. 7.

Stress at Crown

$$k^3 - 3(.5 - \frac{e}{h})k^2 + 12np\frac{e}{h}k = 6np(\frac{e}{h} + 2\frac{a^2}{h^2}) \qquad (27)$$

$$k^3 - 3(.5 - \frac{3.18}{3.67})k^2 + 12 \times 15 \times .0105 \frac{3.18}{3.67} k =$$

$$6 \times 15 \times .0105 (\frac{3.18}{3.67} + 2 \frac{1.50^2}{3.67^2}) \qquad k = .477$$

$$\frac{M}{bf_c h^2} = 1/12k(3 - 2k) + \frac{2pn}{k} \cdot \frac{a^2}{h^2}$$

$$f_c = \frac{275012}{2.5 \times 3.67^2 \times .192 \times 144} = 295\#/\text{sq.in.}$$

$$f_s = nf_c = 15 \times 295 = 4430\#/\text{sq.in.}$$

Stress at Section No. 5

$$k^3 - 3(.5 - \frac{1.78}{3.95})k^2 + 12 \times 15 \times .0098 \frac{1.78}{3.95} k =$$

$$6 \times 15 \times .0098(\frac{1.78}{3.95} + 2 \frac{1.64^2}{3.95^2}) \qquad k = .634$$

$$f_c = \frac{153800}{2.5 \times 3.95^2 \times .172 \times 144} = 159\#/\text{sq.in.}$$

$$f_s = 15 \times 159 = 2385\#/\text{sq.in.}$$

Stress at Section No. 6

$$k^3 - 3(.5 - \frac{.92}{4.1})k^2 + 12 \times 15(.0094)(\frac{.92}{4.1})k =$$

$$6 \times 15 \times .0094(\frac{.92}{4.1} + 2\,\frac{1.72^2}{4.1^2}) \qquad k = .959$$

$$f_c = \frac{79488}{2.5 \times 4.1^2 \times 1.112 \times 144} = 118\#/\text{sq.in.}$$

$$f_s = 15 \times 118 = 1770\#/\text{sq.in.}$$

Stress at Section No. 7

$$12k(1 + 2 \times 15 \times .0090)\,\frac{.20}{4.3} = 1 + \frac{24 \times 15 \times .0090 \times 1.82^2}{4.3^2}$$

$$\frac{6(1 + 2 \times 15 \times .0090)\ .20}{4.3} \qquad k = 2.73$$

$$f_c = \frac{17280}{2.5 \times 4.3^2 \times .0482 \times 144} = 54\#/\text{sq.in.}$$

$$f_s = 15 \times 54 = 810\#/\text{sq.in.}$$

Stress at Springing Line

$$k^3 - 3(.5 - \frac{10.40}{6.17})k^2 + 12 \times 15 \times .0063\,\frac{10.40}{6.17}k =$$

$$6 \times 15 \times .0063(\frac{10.40}{6.17} + 2(\frac{5.50^2}{6.17^2})) \qquad k = .495$$

$$f_c = \frac{898560}{2.5 \times 6.17^2 \times .392 \times 144} = 168\#/\text{sq.in.}$$

$$f_s = 15 \times 168 = 2520\#/\text{sq.in.}$$

Art. 18 Rib Shortening Stresses

The average compressive stress at any section was found by dividing the thrust at that section by the area of the transformed section of arch ring. This is nearly uniform thruout the arch and equal to about 365 lbs./sq.inch.

Average thrust (crown) $= \frac{605000}{bh + n(A + A')}$

" " " $= \frac{605000}{30 \times 44 + 15 \times 13.92} = 402\#/\text{sq.in.}$

" " (Sec. 5) $= \frac{618000}{30 \times 47 + 209} = 383\#/\text{sq.in.}$

" " (Sec. 6) $= \frac{618000}{30 \times 49 + 209} = 368\#/\text{sq.in.}$

" " (Sec. 7) $= \frac{644000}{30 \times 52 + 209} = 364\#/\text{sq.in.}$

" " (Sp. L.) $= \frac{742800}{30 \times 74 + 209} = 306\#/\text{sq.in.}$

Average compressive stress $= 365\#/\text{sq.in.}$

$$H_o = -\frac{I}{ds} \cdot \frac{fcln}{2(n\sum y^2 - (\sum y)^2)}$$

$$H_o = -\frac{365 \times 88.67 \times 10 \times 144}{.22 \times 2(10 \times 222.08 - 31.83^2)}$$

$$H_o = -8650\#$$

This thrust is equal to 10% of the thrust due to temperature change, already found. The resulting moments and stresses will then be 10% of those due to temperature change.

Recapitulation of Stresses

Section	L.L. over 1/2 span	L.L. over entire span	Temp.	Rib Shtg.	Max. f_c	f_s
Crown	470#	570#	295#	30#	895#	13450#
5	473	400	159	16	648	9720
6	458	404	118	12	588	8830
7	421	403	54	5	480	7200
Sp. L.	329	324	168	17	514	7720

Art. 19 Design of Abutments

In the design of the abutment, pier and retaining wall, the combination of an analytical and graphical methods was used. Design sheet number one explains the notation used.

$$W_1 = (7.9+4)/2 \times 3.7 \times 2.5 \times 150 = 8255\#$$

$$W_2 = 1/6(6+6+6)11.7 \times 7.9 \times 150 = \underline{41550}$$

$$W_r = 49805\#$$

Moments about W_2

Distance from W_r to $W_2 = \frac{8255 \times 5.6}{49805} = 0.93'$

$T_R = 698000\#$ $W_R = 49804\#$ $R_R = 740000\#$

R_R acts 1.8 ft. from the center, therefore inside the middle third of the base.

"V" is a component force of final resultant R_R, and it is perpendicular to the base.

"V" = 570000# $X_o = 1.8'$ $\frac{x}{I} = \frac{bd^2}{6}$

Pressure at toe $= \frac{V}{A} + \frac{VX_oX}{I} = \frac{570000}{82} +$

$+ \frac{570000 \times 1.8 \times 6}{6 \times 13^2} = 13010\#/\text{sq.ft.}$

Pressure at heel $= \frac{V}{A} - \frac{VXoX}{I} = 890\#/\text{sq.ft.}$

Allowable pressure 40000#/sq.ft.

Art. 20 Design of Pier

Weight of pier = W = 48213#

D. L. Thrust = T = 610000#

Full L. L. Thrust = T' = 820000#

Resultant Thrust = T_R = 780000#

Combining W and T_R gives R_R = 660000#

R_R acts within the middle third of the base.

Wind loads acting at intersection of arch axes.

$$6 \times 85 \times 30 = 15300\#$$

$$30 \times 85 = \underline{25500\#}$$

Total Wind Load = 40800#

Ice thrust acting 4 ft. above box of pier

$$900 \times 8 \times 4 = 28800\#$$

Combining the wind and ice loads gives X_R = 69600, acting .52 ft. below wind load.

Combining X_R and R_R gives the final resultant of 665000# acting inside the middle third of the base.

$V = R_R = 660000 \quad x_o = .5' \quad x = 4.3' \quad I = 503$

$$\text{Pressure at toe} = \frac{660000}{66} + \frac{660000 \times .5 \times 4.3}{503}$$

$$= 12820\#/\text{sq.ft.}$$

$$\text{Pressure at Heel} = \frac{660000}{66} - \frac{660000 \times .5 \times 4.3}{503} =$$

$$= 7180\#/\text{sq.ft.}$$

(40000#/sq.ft. is allowed)

Art. 21 Design of Retaining Wall

Gravity type shall be used and it will be designed for a fluid weight of 25 lbs. per cu.

ft.

Floor system load on wall $= 27075\#$

Wall load $W_1 = \frac{2+5}{2} \times 21 \times 150 = 11025$

" " $W_2 = \frac{1.5+2}{2} \times 3 \times 150 = 788$

Fluid load $W_3 = \frac{3 \times 2}{2} \times 25 = \underline{788}$

Total Load $W_R = 39676\#$

Moments about W_1

$788 \times 3.2 + 27075 \times 0.8 - 788 \times 2.3 = 39676x$

Distance of W_R from $W_1 = x = 0.51'$

Fluid Pressure $P = \frac{25 \times 21 \times 21}{2} = 5513\#$

Final Resultant $R_R = 40000\#$

Pressure at toe $= \frac{40000}{8 \times 1} + \frac{40000 \times 0.6 \times 6}{8 \times 8} = 7250\#/\text{sq.ft.}$

Pressure at heel $= \frac{40000}{8 \times 1} - \frac{40000 \times 0.6 \times 6}{8 \times 8} = 2750\#/\text{sq.ft.}$

Cantilever. Span = 3' 0"

Shear at inner end $= \frac{7250+5750}{2} \times 3 - 788 = 18712\#$

$V = \frac{18712}{20 \times 12} = 78\#/\text{sq.in.}$

3/8" stirrups will be used, spaced 4' centers.

B. M. at inner end $= -(\frac{7250+5750}{2})1.6 + 788 \times 1.4$

" " " " $= 5375'\# = 64740''\#$

$$A = \frac{64740}{16000 \times .88 \times 20} = .23 \text{ sq.in.}$$

1" dia. 3" c. to c.

$$u = \frac{18712}{12/3 \times 2.36 \times .88 \times 20} = 84\#/\text{sq.in.}$$

3/8" dia. rods spaced 24" shall be used to prevent shrinkage and temperature cracks.

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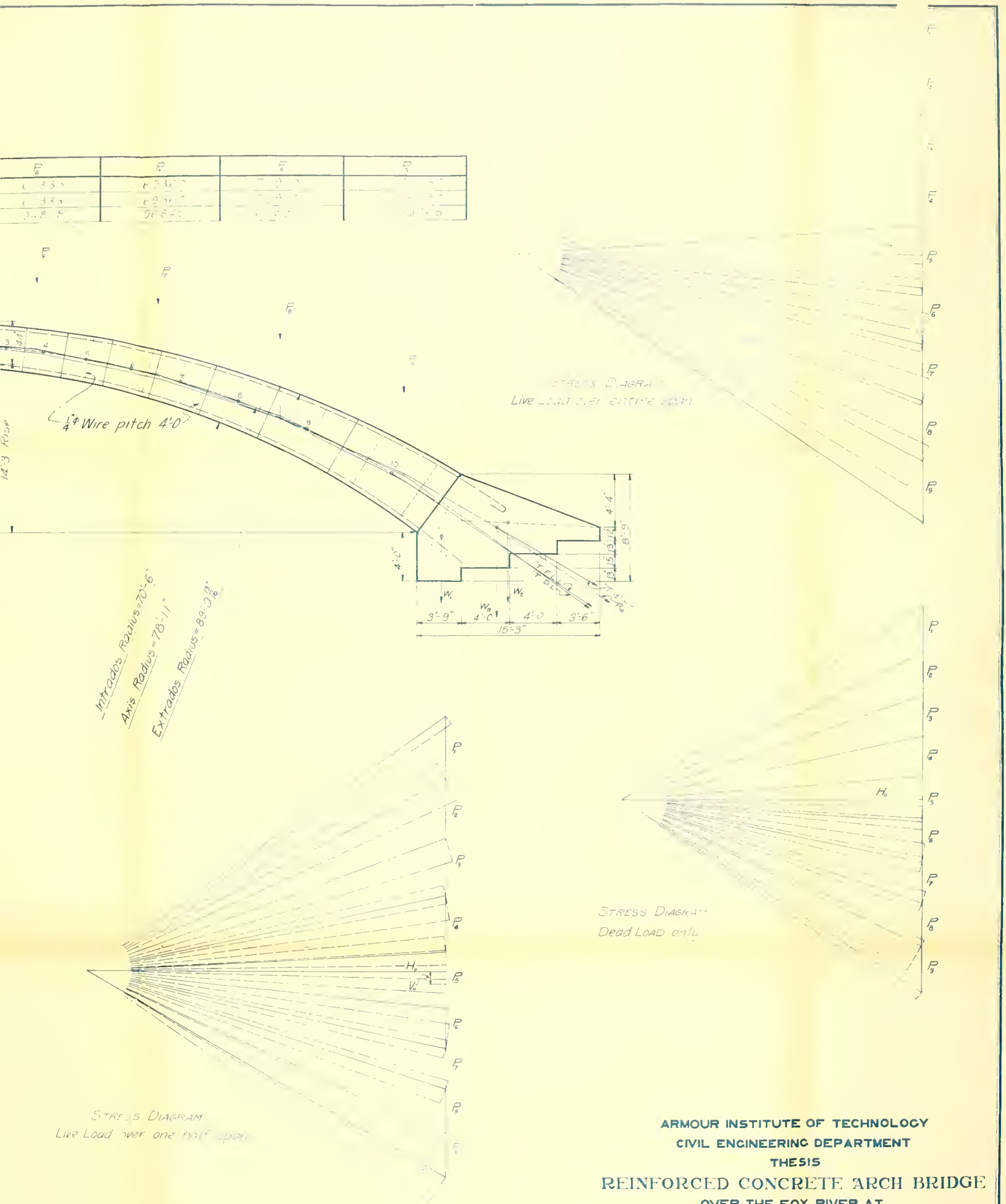
P_6	P_7	P_8	P_9
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]



ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT
THESIS

REINFORCED CONCRETE ARCH BRIDGE

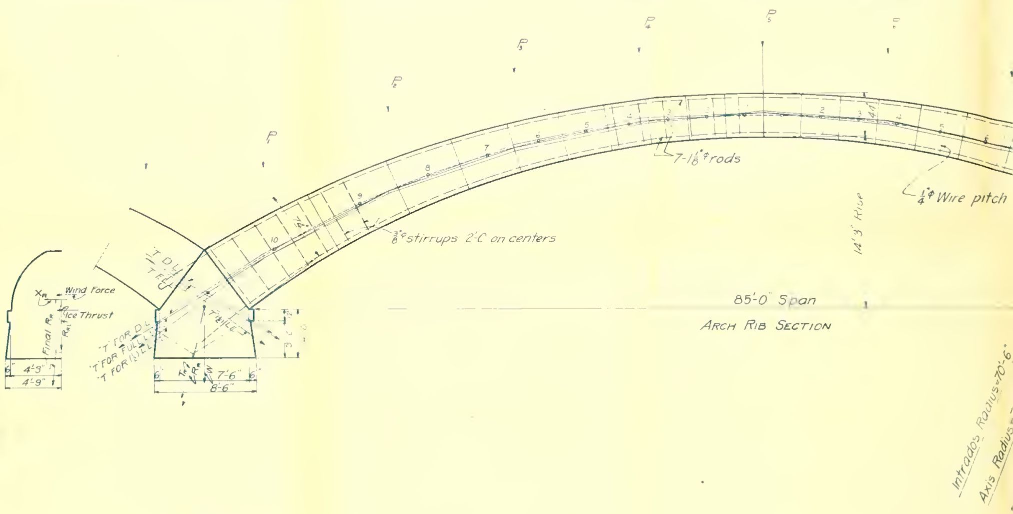
OVER THE FOX RIVER AT
MONTGOMERY KANE COUNTY ILLINOIS
DESIGN

Scale $\frac{1}{4}$"=1'-0"

May 1911

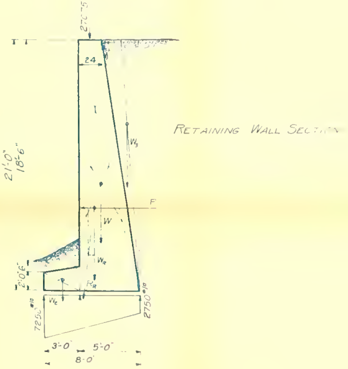
Joseph L. Duffy

LOADINGS	P	P_2	P_3	P_4	P_5	P_6	P_7
Dead Load	78280	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Live Load over one-half span	105765	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Live Load over entire span	105765	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]



85'-0" Span

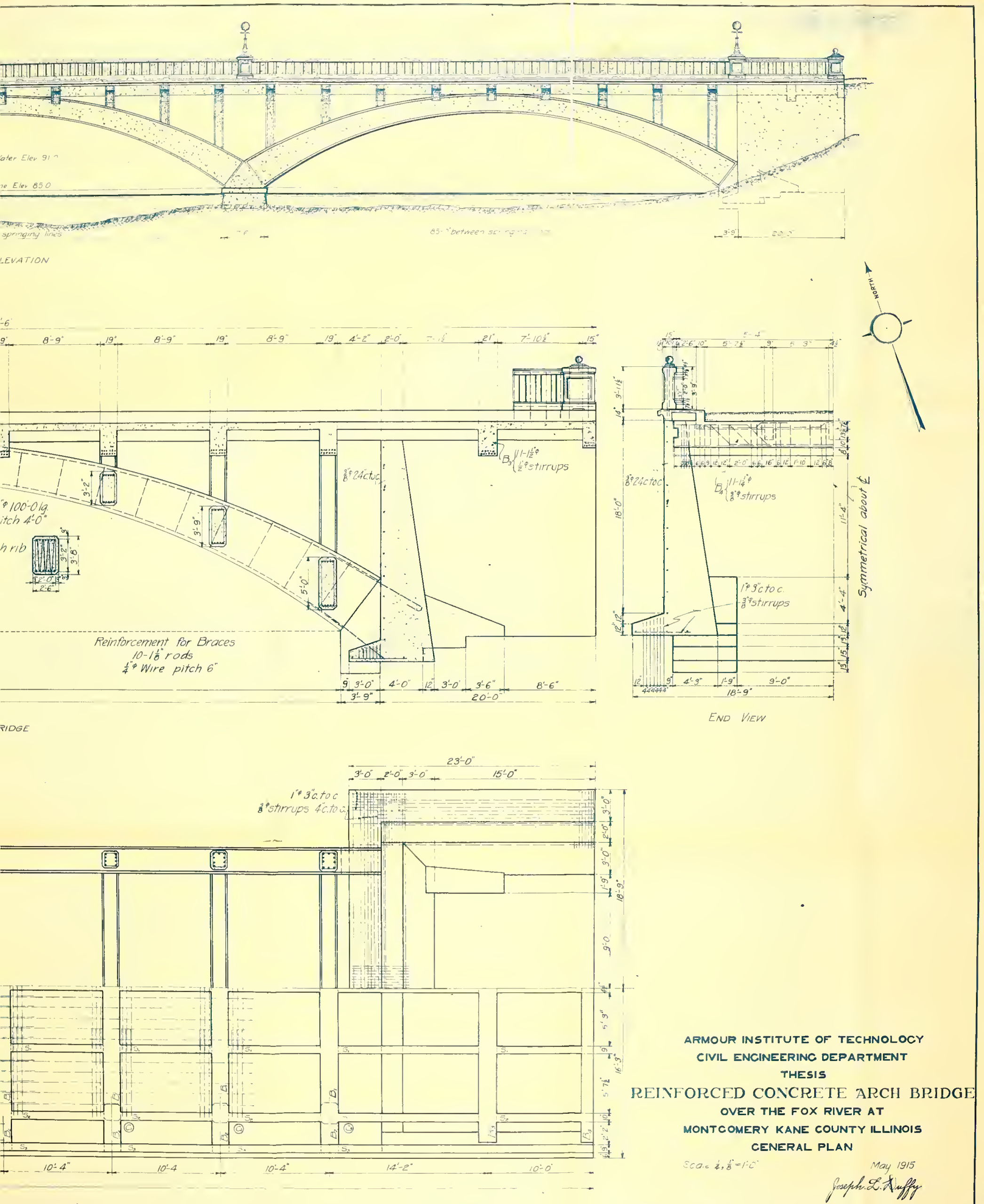
ARCH RIB SECTION



RETAINING WALL SECTION

STRESS D

Live Load over



ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT
THESIS

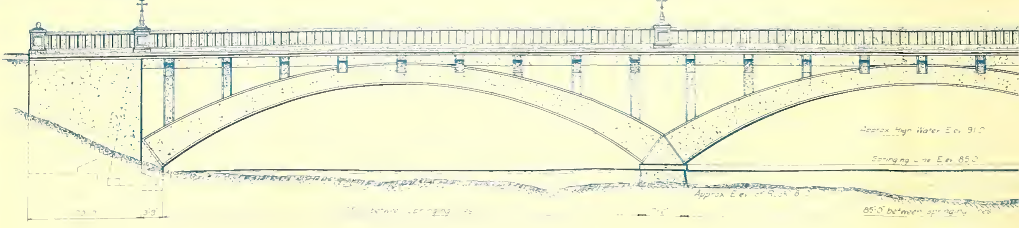
REINFORCED CONCRETE ARCH BRIDGE

OVER THE FOX RIVER AT
MONTGOMERY KANE COUNTY ILLINOIS
GENERAL PLAN

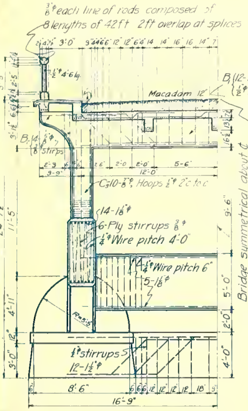
Scale ¼, ⅛" = 1'-0"

May 1915

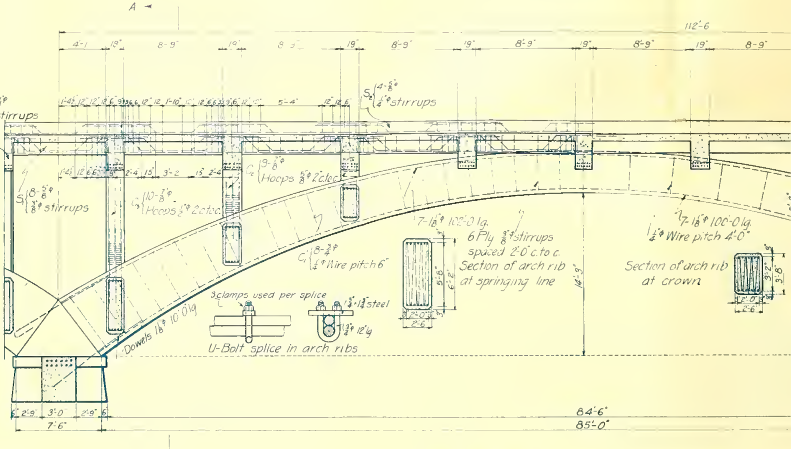
Joseph L. Duffy


Approx. High Water Elev. 91.0
Springing Line Elev. 85.0
Approx. Elev. of Rock
85'-0" between springing lines
3'-9"

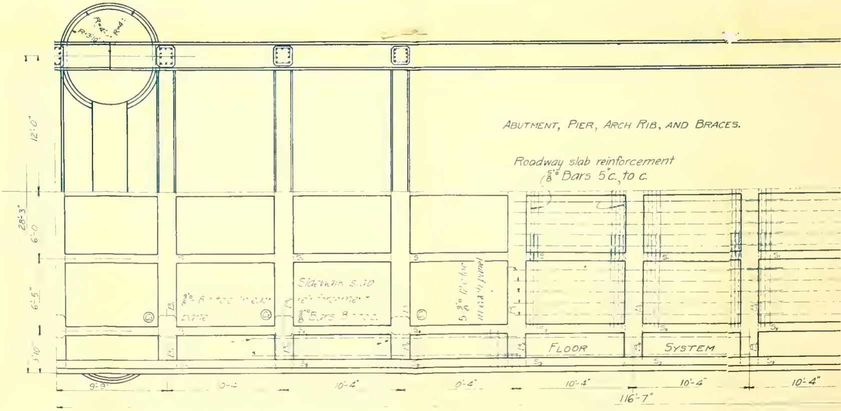
GENERAL ELEVATION



SECTION A-A



LONGITUDINAL SECTION AT CENTER LINE OF BRIDGE


ABUTMENT, PIER, ARCH RIB, AND BRACES.
Roadway slab reinforcement
5/8" Bars 5" c. to c.
Sidewalk slab reinforcement
3/8" Bars 8" c. to c.
5 3/8" 12" c. to c. in each panel
FLOOR
SYSTEM
R=4'
R=3'10"
12'-0"
28'-3"
6'-0"
6'-5"
3'-10"
9'-9"
10'-4"
10'-4"
116'-7"

PLAN